This procedure lists the available design aids and discusses the theoretical concepts needed to hydraulically design a storm sewer system operating under full flow and pressure flow conditions. In addition, criteria for pipe diameter strength, alignment, and flow line depth are discussed under the heading " Standards for Storm Drain Pipes."

Further discussions on storm sewer design may be found in the ASCE book entitled "Design and Construction of Sanitary and Storm Sewers."(1)

Design Aids

The first flow friction formula used to design closed conduits (partially full, full, or pressure flow) and open channels was published by Kutter in 1869 and is known as Kutter's Formula. Since then, additional flow friction formulas that have gained widely acceptable usage are (1) the Darcy-Weisbach Equation, (2) the Manning Formula, and (3) the Hazen-Williams Formula.

Because of its simplicity, the Manning Formula is used by the Department of Transportation for the design of closed conduits under partially full, full, or pressure flow conditions. For the Manning Formula, the full flow capacity of a specific pipe size is a function of pipe slope and roughness coefficient (Manning's n equals Kutter's n) (see Figure 1).

The design of closed conduits in a partially full flow condition through the direct application of the Manning Formula can be accomplished only through trial and error. However, faster design of closed conduits in partially full flow, full flow, or pressure flow may be accomplished through the use of one or more of the following design aids:

1. Circular pipe flow charts - Bureau of Public Roads, "Design Charts for Open-Channel Flow," Hydraulic Design Series No. 3, Charts 35 to 52. (5)
2. A nomograph for the solution of the Manning Formula in conjunction with a graph of hydraulic elements for a circular section - See Figures 2 and 3, respectively.
3. Nomographs for the direct solution of pipe flow - See Figures 4 and 5.
4. Slide rules - Solution of Manning Formula, copyright 1973, American Concrete Pipe Association; and Solution of Kutter's Formula, copyright 1947, 1961, Irving Goldfein, Civil Engineer, Bureau of Engineers, Municipal Building, Milwaukee, Wisconsin.

Conduit Design - Full Flow

Tentatively, the pipe gradient is set equal to the pavement gradient, then a pipe size is selected that approximately equals the design flow under full flow conditions. Generally, no standard size pipe will carry the design flow exactly at full depth flow. Therefore, the next larger size pipe must be selected, the pipe gradient modified, or both.

Some publications state that storm sewer pipes should be designed for a 0.8 full flow condition. However, the capacity of a pipe is the same at a 0.8 full flow condition as at a full flow condition, and hence either design method will produce the same required pipe size. Although the capacity of a pipe is largest between a 0.8 full flow condition and a full flow condition, pipes should never be designed for flow in this very unstable, unpredictable flow region.

Under special conditions, such as connecting to an existing undersized storm sewer system, or backwater from a receiving stream, etc., pipes may be allowed to operate under pressure, provided the hydraulic head does not cause any pavement flooding or property damage.

For normal full flow pipe design, no allowance need be or shall be made for energy losses at bends, joints, and transitions, unless anticipated high-energy losses could cause flooding problems. When pressure flow conditions are encountered, the system must be designed for the high-energy losses produced by the pressure-induced high flow velocities. Too large a pressure flow can cause pavement flooding, basement flooding, manhole cover popping, etc.

Pressure Flow

Storm sewer systems operating under pressure flow must be designed for energy losses (head losses). These energy losses are used to determine the energy grade line and the hydraulic grade line of a storm sewer system. This section briefly explains these hydraulic concepts.

There are six categories of energy loss that should be considered. They are:

Manhole losses	Exit losses
Inlet losses	Bend losses
Entrance losses	Friction losses

Manhole Losses

Manhole losses may be determined by using the procedure presented in the "Urban Drainage Design" (4). The manhole loss coefficient for storm drain design can be evaluated by K x ($V_o^2/2g$) where K can be approximated by:

$$\mathbf{K = K_o C_D C_d C_Q C_p C_B}$$

where:

K = adjusted loss coefficient

K_o= initial head loss coefficient based on relative manhole size.

C_D= correction factor for pipe diameter (pressure flow only)

C_d= correction factor for flow depth (non-pressure flow only)

C_Q= correction factor for relative flow.

C_B= correction factor for benching.

C_p= correction factor for plunging flow.

A discussion follows on each of the correction factors.

Relative Manhole Size: K_o is estimated as a function of the relative manhole size and the angle of deflection between the inflow and outflow pipes.

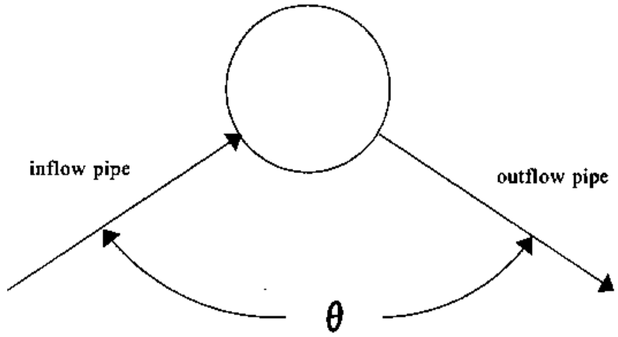
$$K_o = 0.1\ (b/D_o)(1-\sin Æ) + 1.4\ (b/D_o)^{0.15} \sin Æ$$

where:

Æ = the angle between the inflow and outflow pipes

b = manhole diameter

D_o = outlet pipe diameter



Pipe Diameter: The correction factor for pipe diameter is significant only in pressure flow situations when the ratio of the water depth in the manhole(d) to the outlet pipe diameter (D_o), d/D_o, is greater than 3.2.

$$C_D = (D_o/D_i)^3$$

where:
D_i = incoming pipe diameter
D_o = outgoing pipe diameter

Flow Depth: The correction factor is significant only in cases of free surface flow or low pressures, when d/D_o ratio is less than 3.2 and is only applied in such cases. The water depth in the manhole is approximated as the level of the hydraulic grade line at the upstream end of the outpipe. The correction factor for flow depth, C_d is calculated by:

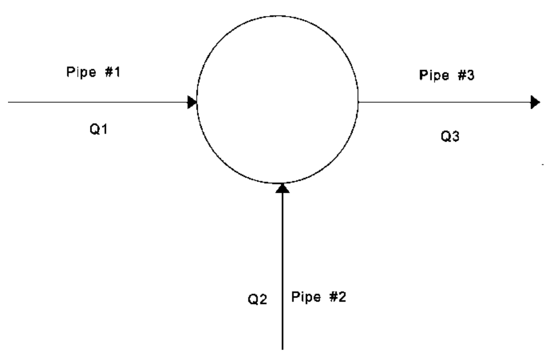
$$C_d = 0.5\ (d/D_o)^{0.6}$$

where:
d = water depth in manhole above outlet pipe
D_o = outlet pipe diameter

Relative Flow: The correction factor for relative flow, C_Q, is a function of the percentage of flow coming in through the pipe of interest as well as the angle of the incoming flow versus other incoming pipes. It is calculated by the following:

$$C_Q = (1\text{-}2 \sin Æ) \times [1 - (Q_i/Q_o)]^{0.75} + 1$$

where:
C_Q = correction factor for relative flow
Æ = the angle between the inflow and outflow pipes.
Q_i = flow in the inflow pipe
Q_o = flow in the outflow pipe



The example below illustrates two situations to determine the impact of pipe #2 entering the manhole.

Example 1
Q_1 = 6 cfs, Q_2 = 3 cfs
Q_3 = 9 cfs then

$C_Q = (1\text{-}2 \sin 180°) \times [1 - (6/9)]^{.75} + 1$
$= 1.44$

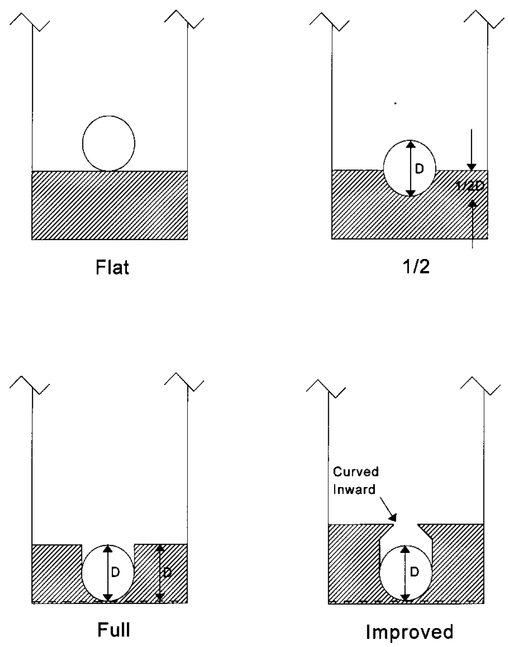
Example 2

Q_1 = 3 cfs, Q_1 = 6 cfs
Q_3 = 9 cfs then

$C_Q = (1\text{-}2 \sin 180°) \times [1 - (3/9)]^{.75} + 1$
$= 1.74$

** Free surface flow, $d/D_o < 1.0$

A linear interpolation is performed for flow depths between the submerged and unsubmerged conditions. The following schematic shows each of the four conditions described above.



To estimate the head losses through a manhole from the outlet pipe to a particular inlet pipe, multiply the correction factors together to get the head loss coefficient, K. Then, multiply K by the velocity head in the outflow pipe to estimate the minor loss for the connection.

Manhole losses may also be determined by using the design methodologies, design charts and examples of pressure flow design given in the University of Missouri Bulletin entitled "Pressure Changes at Storm Drain Junctions."(2)

Inlet Losses

Inlet losses may be determined by using the design methodologies, design charts and examples of pressure flow design given in the University of Missouri Bulletin entitled "Pressure Changes at Storm Drain Junctions."(2)

Entrance, Exit, and Bend Losses

The general equation for these losses, expressed as a function of pipe flow velocities, is:

$$H = K V^2/2g$$

where:
H = head loss
K = loss coefficient
V = average pipe velocity
g = 32.2 ft/sec^2

Entrance losses need only be considered when the storm sewer originates at a culvert. Entrance loss coefficients Ke for various entrance conditions can be obtained from HDS #5, (3) Table 12.

Exit losses for sewer pipe discharging into a receiving stream will produce an energy loss at its outlet equivalent to one velocity head; K equals 1.0.

Bend loss coefficient K_b values for curvilinear and miter bends can be obtained from Figures 6 and 7.

Friction Losses

The largest losses in a storm sewer system are friction losses. They are directly related to the velocity in the pipe and hence the higher the velocity, the greater the friction loss and vice versa. The slope of the friction loss can be estimated by using Figures 4 (corrugated metal pipe) and 5 (concrete).

The total frictional head loss in a given length of pipe can be computed with the following equation:

$$H_f = S_f L$$

where:

H_f= head loss for friction

S_f= slope of the energy grade line

L = length of the conduit

Energy and Hydraulic Grade Lines (EGL and HGL)

The energy grade line shows the total energy at any point in a storm sewer, whereas the hydraulic grade line shows the pressure head or the water surface level in open tubes if they are inserted in the pipe. The EGL must always drop in the direction of flow; however, the HGL may rise at hydraulic structures, such as manholes.

The EGL and the HGL might need to be calculated when part of the storm sewer system might be operating under pressure whether or not the outfall is submerged. These computations are made starting at the outfall where the EGL and HGL coincide at the water surface of the discharge pond.

The EGL for a storm sewer is determined by adding the energy losses in a progressive manner from the outfall to the upper end of the system. The elevation of the HGL can be determined by subtracting from the elevation of the EGL the value of the velocity head ($V^2/2g$) for each individual pipe.

For a step-by-step methodology of the determination of the EGL and HGL for surcharged full flow, see Procedure 13-25-45.

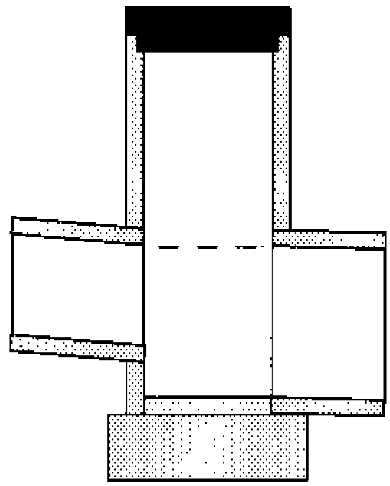
Hydraulic Standards for Storm Drain Pipe

Minimum Pipe Slope

Minimum full flow velocity shall be 2.5 fps, and preferably 3 fps, to prevent deposition of solids. If the design flow rate based on future conditions is appreciably larger than the present flow rate, it may be advisable that the minimum pipe slope be checked with the present flow rate. Desirable full flow velocity shall be 10 to 15 fps. For some standard size concrete pipe (n = 0.013), the minimum slopes required to maintain a self-cleaning velocity of 2.5 or 3.0 fps at full flow are as follows:

Pipe Diameter (Inches)	Minimum Slope (Ft./Ft.)	
	2.5 fps	3.0 fps
12	.0030	.0044
15	.0023	.0032
18	.0018	.0025
24	.0012	.0017

In the majority of cases, the flow line depth is determined by the conduit size and the slope requirements. However, additional factors, such as hydraulic grade line elevations, lateral connections, vertical clearance of obstructions, etc., may also, in certain cases, control the required flow line depth. Moreover, the flow line depth of the conduit should be set to maintain the calculated hydraulic grade line (water surface elevation) at inlets, junction chambers, and manholes at one foot (**300 mm**) or more below the grate or cover. If practicable, the crowns of pipes connecting to inlets, junctions, and manholes should be held at the same elevation. See the sketch below.



REFERENCES

(1)American Society of Civil Engineers and Water Pollution Control Federation, Design and Construction of Sanitary and Storm Sewers, ASCE No. 37 or WPCF No. 9, New York, New York, 1991, 332 pp.

(2)(2)Sangster, W.M., Wood, H.W., Smerdon, E.T., and Bossy, H.G., "Pressure Changes at Storm Drainage Junctions," University of Missouri, Engineering Experiment Station Bulletin 41, 1958, 132 pp.

(3)U.S. Department of Transportation, Federal Highway Administration, Hydraulic Design of Highway Culverts, Hydraulic Design Series No. 5, September 1985.

(4)U.S. Department of Transportation, Federal Highway Administration, Urban Drainage Design, Publication No. FHWA-HI-89-035, April 1989.

(5) (5)U.S. Department of Transportation, Federal Highway Administration, Design Chart for Open - Channel Flow, Hydraulic Design Series No. 3, August 1961.

★

Table 1.—Manning roughness coefficients, *n* [1]

	Manning's *n* range [2]
I. Closed conduits:	
A. Concrete pipe	0.011–0.013
B. Corrugated-metal pipe or pipe-arch:	
1. 2⅔ by ½-in. corrugation (riveted pipe): [3]	
a. Plain or fully coated	0.024
b. Paved invert (range values are for 25 and 50 percent of circumference paved):	
(1) Flow full depth	0.021–0.018
(2) Flow 0.8 depth	0.021–0.016
(3) Flow 0.6 depth	0.019–0.013
2. 6 by 2-in. corrugation (field bolted)	0.03
C. Vitrified clay pipe	0.012–0.014
D. Cast-iron pipe, uncoated	0.013
E. Steel pipe	0.009–0.011
F. Brick	0.014–0.017
G. Monolithic concrete:	
1. Wood forms, rough	0.015–0.017
2. Wood forms, smooth	0.012–0.014
3. Steel forms	0.012–0.013
H. Cemented rubble masonry walls:	
1. Concrete floor and top	0.017–0.022
2. Natural floor	0.019–0.025
I. Laminated treated wood	0.015–0.017
J. Vitrified clay liner plates	0.015
II. Open channels, lined [4] (straight alinement): [5]	
A. Concrete, with surfaces as indicated:	
1. Formed, no finish	0.013–0.017
2. Trowel finish	0.012–0.014
3. Float finish	0.013–0.015
4. Float finish, some gravel on bottom	0.015–0.017
5. Gunite, good section	0.016–0.019
6. Gunite, wavy section	0.018–0.022
B. Concrete, bottom float finished, sides as indicated:	
1. Dressed stone in mortar	0.015–0.017
2. Random stone in mortar	0.017–0.020
3. Cement rubble masonry	0.020–0.025
4. Cement rubble masonry, plastered	0.016–0.020
5. Dry rubble (riprap)	0.020–0.030
C. Gravel bottom, sides as indicated:	
1. Formed concrete	0.017–0.020
2. Random stone in mortar	0.020–0.023
3. Dry rubble (riprap)	0.023–0.033
D. Brick	0.014–0.017
E. Asphalt:	
1. Smooth	0.013
2. Rough	0.016
F. Wood, planed, clean	0.011–0.013
G. Concrete-lined excavated rock:	
1. Good section	0.017–0.020
2. Irregular section	0.022–0.027
III. Open channels, excavated [4] (straight alinement,[5] natural lining):	
A. Earth, uniform section:	
1. Clean, recently completed	0.016–0.018
2. Clean, after weathering	0.018–0.020
3. With short grass, few weeds	0.022–0.027
4. In gravelly soil, uniform section, clean	0.022–0.025
B. Earth, fairly uniform section:	
1. No vegetation	0.022–0.025
2. Grass, some weeds	0.025–0.030
3. Dense weeds or aquatic plants in deep channels	0.030–0.035
4. Sides clean, gravel bottom	0.025–0.030
5. Sides clean, cobble bottom	0.030–0.040
C. Dragline excavated or dredged:	
1. No vegetation	0.028–0.033
2. Light brush on banks	0.035–0.050
D. Rock:	
1. Based on design section	0.035
2. Based on actual mean section:	
a. Smooth and uniform	0.035–0.040
b. Jagged and irregular	0.040–0.045
E. Channels not maintained, weeds and brush uncut:	
1. Dense weeds, high as flow depth	0.08–0.12
2. Clean bottom, brush on sides	0.05–0.08
3. Clean bottom, brush on sides, highest stage of flow	0.07–0.11
4. Dense brush, high stage	0.10–0.14
IV. Highway channels and swales with maintained vegetation [6,7] (values shown are for velocities of 2 and 6 f.p.s.):	
A. Depth of flow up to 0.7 foot:	
1. Bermudagrass, Kentucky bluegrass, buffalograss:	
a. Mowed to 2 inches	0.07–0.045
b. Length 4–6 inches	0.09–0.05
2. Good stand, any grass:	
a. Length about 12 inches	0.18–0.09
b. Length about 24 inches	0.30–0.15
3. Fair stand, any grass:	
a. Length about 12 inches	0.14–0.08
b. Length about 24 inches	0.25–0.13
B. Depth of flow 0.7–1.5 feet:	
1. Bermudagrass, Kentucky bluegrass, buffalograss:	
a. Mowed to 2 inches	0.05–0.035
b. Length 4 to 6 inches	0.06–0.04
2. Good stand, any grass:	
a. Length about 12 inches	0.12–0.07
b. Length about 24 inches	0.20–0.10
3. Fair stand, any grass:	
a. Length about 12 inches	0.10–0.06
b. Length about 24 inches	0.17–0.09
V. Street and expressway gutters:	
A. Concrete gutter, troweled finish	0.012
B. Asphalt pavement:	
1. Smooth texture	0.013
2. Rough texture	0.016
C. Concrete gutter with asphalt pavement:	
1. Smooth	0.013
2. Rough	0.015
D. Concrete pavement:	
1. Float finish	0.014
2. Broom finish	0.016
E. For gutters with small slope, where sediment may accumulate, increase above values of *n* by	*0.002*
VI. Natural stream channels:[8]	
A. Minor streams [9] (surface width at flood stage less than 100 ft.):	
1. Fairly regular section:	
a. Some grass and weeds, little or no brush	0.030–0.035
b. Dense growth of weeds, depth of flow materially greater than weed height	0.035–0.05
c. Some weeds, light brush on banks	0.035–0.05
d. Some weeds, heavy brush on banks	0.05–0.07
e. Some weeds, dense willows on banks	0.06–0.08
f. For trees within channel, with branches submerged at high stage, increase all above values by	*0.01–0.02*
2. Irregular sections, with pools, slight channel meander; increase values given in 1a–e about	*0.01–0.02*
3. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage:	
a. Bottom of gravel, cobbles, and few boulders	0.04–0.05
b. Bottom of cobbles, with large boulders	0.05–0.07
B. Flood plains (adjacent to natural streams):	
1. Pasture, no brush:	
a. Short grass	0.030–0.035
b. High grass	0.035–0.05
2. Cultivated areas:	
a. No crop	0.03–0.04
b. Mature row crops	0.035–0.045
c. Mature field crops	0.04–0.05
3. Heavy weeds, scattered brush	0.05–0.07
4. Light brush and trees: [10]	
a. Winter	0.05–0.06
b. Summer	0.06–0.08
5. Medium to dense brush: [10]	
a. Winter	0.07–0.11
b. Summer	0.10–0.16
6. Dense willows, summer, not bent over by current	0.15–0.20
7. Cleared land with tree stumps, 100–150 per acre:	
a. No sprouts	0.04–0.05
b. With heavy growth of sprouts	0.06–0.08
8. Heavy stand of timber, a few down trees, little undergrowth:	
a. Flood depth below branches	0.10–0.12
b. Flood depth reaches branches	0.12–0.16
C. Major streams (surface width at flood stage more than 100 ft.): Roughness coefficient is usually less than for minor streams of similar description on account of less effective resistance offered by irregular banks or vegetation on banks. Values of *n* may be somewhat reduced. Follow recommendation in publication cited [8] if possible. The value of *n* for larger streams of most regular section, with no boulders or brush, may be in the range of	0.028–0.033

from Hydraulic Design Series No. 3, "Design Charts for Open-Channel Flow"

Footnotes to Table 1 appear on page 2 of this figure

Footnotes to Table 1

[1] Estimates are by Bureau of Public Roads unless otherwise noted.

[2] Ranges indicated for closed conduits and for open channels, lined or excavated, are for good to fair construction (unless otherwise stated). For poor quality construction, use larger values of n.

[3] *Friction Factors in Corrugated Metal Pipe*, by M. J. Webster and L. R. Metcalf, Corps of Engineers, Department of the Army; published in Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, vol. 85, No. HY9, Sept. 1959, Paper No. 2148, pp. 35–67.

[4] For important work and where accurate determination of water profiles is necessary, the designer is urged to consult the following references and to select n by comparison of the specific conditions with the channels tested:

Flow of Water in Irrigation and Similar Channels, by F. C. Scobey, Division of Irrigation, Soil Conservation Service, U.S. Department of Agriculture, Tech. Bull. No. 652, Feb. 1939; and

Flow of Water in Drainage Channels, by C. E. Ramser, Division of Agricultural Engineering, Bureau of Public Roads, U.S. Department of Agriculture, Tech. Bull. No. 129, Nov. 1929.

[5] With channel of an alinement other than straight, loss of head by resistance forces will be increased. A small increase in value of n may be made, to allow for the additional loss of energy.

[6] *Handbook of Channel Design for Soil and Water Conservation*, prepared by the Stillwater Outdoor Hydraulic Laboratory in cooperation with the Oklahoma Agricultural Experiment Station; published by the Soil Conservation Service, U.S. Department of Agriculture, Publ. No. SCS-TP-61, Mar. 1947, rev. June 1954.

[7] *Flow of Water in Channels Protected by Vegetative Linings*, by W. O. Ree and V. J. Palmer, Division of Drainage and Water Control, Research, Soil Conservation Service, U.S. Department of Agriculture, Tech. Bull. No. 967, Feb. 1949.

[8] For calculation of stage or discharge in natural stream channels, it is recommended that the designer consult the local District Office of the Surface Water Branch of the U.S. Geological Survey, to obtain data regarding values of n applicable to streams of any specific locality. Where this procedure is not followed, the table may be used as a guide. The values of n tabulated have been derived from data reported by C. E. Ramser (see footnote 4) and from other incomplete data.

[9] The tentative values of n cited are principally derived from measurements made on fairly short but straight reaches of natural streams. Where slopes calculated from flood elevations along a considerable length of channel, involving meanders and bends, are to be used in velocity calculations by the Manning formula, the value of n must be increased to provide for the additional loss of energy caused by bends. The increase may be in the range of perhaps 3 to 15 percent.

[10] The presence of foliage on trees and brush under flood stage will materially increase the value of n. Therefore, roughness coefficients for vegetation in leaf will be larger than for bare branches. For trees in channel or on banks, and for brush on banks where submergence of branches increases with depth of flow, n will increase with rising stage.

Table 2.—Permissible velocities for channels with erodible linings, based on uniform flow in continuously wet, aged channels [1]

Soil type or lining (earth; no vegetation)	Maximum permissible velocities for—		
	Clear water	Water carrying fine silts	Water carrying sand and gravel
	F.p.s.	*F.p.s.*	*F.p.s.*
Fine sand (noncolloidal)	1.5	2.5	1.5
Sandy loam (noncolloidal)	1.7	2.5	2.0
Silt loam (noncolloidal)	2.0	3.0	2.0
Ordinary firm loam	2.5	3.5	2.2
Volcanic ash	2.5	3.5	2.0
Fine gravel	2.5	5.0	3.7
Stiff clay (very colloidal)	3.7	5.0	3.0
Graded, loam to cobbles (noncolloidal)	3.7	5.0	5.0
Graded, silt to cobbles (colloidal)	4.0	5.5	5.0
Alluvial silts (noncolloidal)	2.0	3.5	2.0
Alluvial silts (colloidal)	3.7	5.0	3.0
Coarse gravel (noncolloidal)	4.0	6.0	6.5
Cobbles and shingles	5.0	5.5	6.5
Shales and hard pans	6.0	6.0	5.0

[1] As recommended by Special Committee on Irrigation Research, American Society of Civil Engineers, 1926.

Table 3.—Permissible velocities for channels lined with uniform stands of various grass covers, well maintained [1] [2]

Cover	Slope range	Permissible velocity on—	
		Erosion resistant soils	Easily eroded soils
	Percent	*F.p.s.*	*F.p.s.*
Bermudagrass	0–5 5–10 Over 10	8 7 6	6 5 4
Buffalograss Kentucky bluegrass Smooth brome Blue grama	0–5 5–10 Over 10	7 6 5	5 4 3
Grass mixture	0–5 5–10	5 4	4 3
Lespedeza sericea Weeping lovegrass Yellow bluestem Kudzu Alfalfa Crabgrass	0–5	3.5	2.5
Common lespedeza [3] Sudangrass [3]	[4] 0–5	3.5	2.5

[1] From *Handbook of Channel Design for Soil and Water Conservation* (see footnote 6, table 1, above).

[2] Use velocities over 5 f.p.s. only where good covers and proper maintenance can be obtained.

[3] Annuals, used on mild slopes or as temporary protection until permanent covers are established.

[4] Use on slopes steeper than 5 percent is not recommended.

Table 4.—Factors for adjustment of discharge to allow for increased resistance caused by friction against the top of a closed rectangular conduit [1]

D/B	Factor
1.00	1.21
.80	1.24
.75	1.25
.667	1.27
.60	1.28
.50	1.31
.40	1.34

[1] Interpolations may be made.

Table 5.—Guide to selection of retardance curve

Average length of vegetation	Retardance curve for—	
	Good stand	Fair stand
6–10 inches	C	D.
2–6 inches	D	D.

from Hydraulic Design Series No. 3, "Design Charts for Open-Channel Flow"

Graphic Solution of the Manning Equation

FIGURE 2 is a nomograph for the solution of the Manning equation:

$$V=\frac{1.49}{n}R^{2/3}S^{1/2}.$$

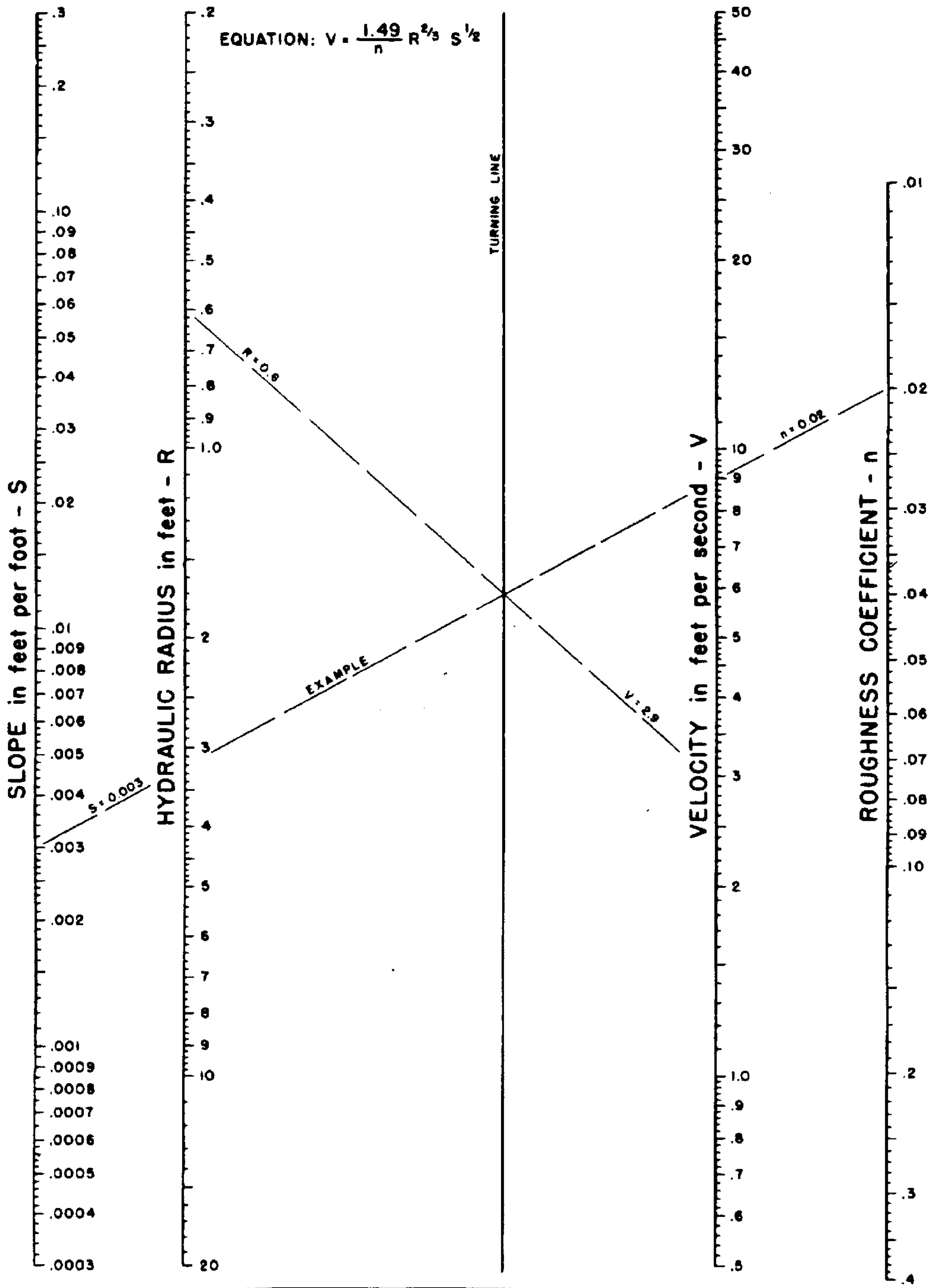
This chart will be found useful when an open-channel flow chart is not available for the particular channel cross section under consideration. Values of n will be found in table 1, and slope S and hydraulic radius $R=A/WP$, where A is the area of cross section and WP is the wetted perimeter, are dimensions of the channel.

Use of the chart is demonstrated by the example shown on the chart itself. Given is a channel with rectangular cross section, 6 feet wide, flowing at a depth of 0.75 foot, with a 0.3-percent slope ($S=0.003$), and $n=0.02$. Area $A=6\times 0.75=4.50$ sq. ft.; wetted perimeter $WP=6+2\times 0.75=7.50$ ft.; then $R=A/WP=4.50/7.50=0.6$.

A straight line is laid on the chart, connecting $S=0.003$ and $n=0.02$. Another straight line is then laid on the chart, connecting $R=0.6$ and the intersection of the first line and the "turning line," and extending to the velocity scale. Reading this scale, $V=2.9$.

The chart may, of course, be used to find any one of the four values represented, given the other three; and may also be used for channels with cross sections other than rectangular.

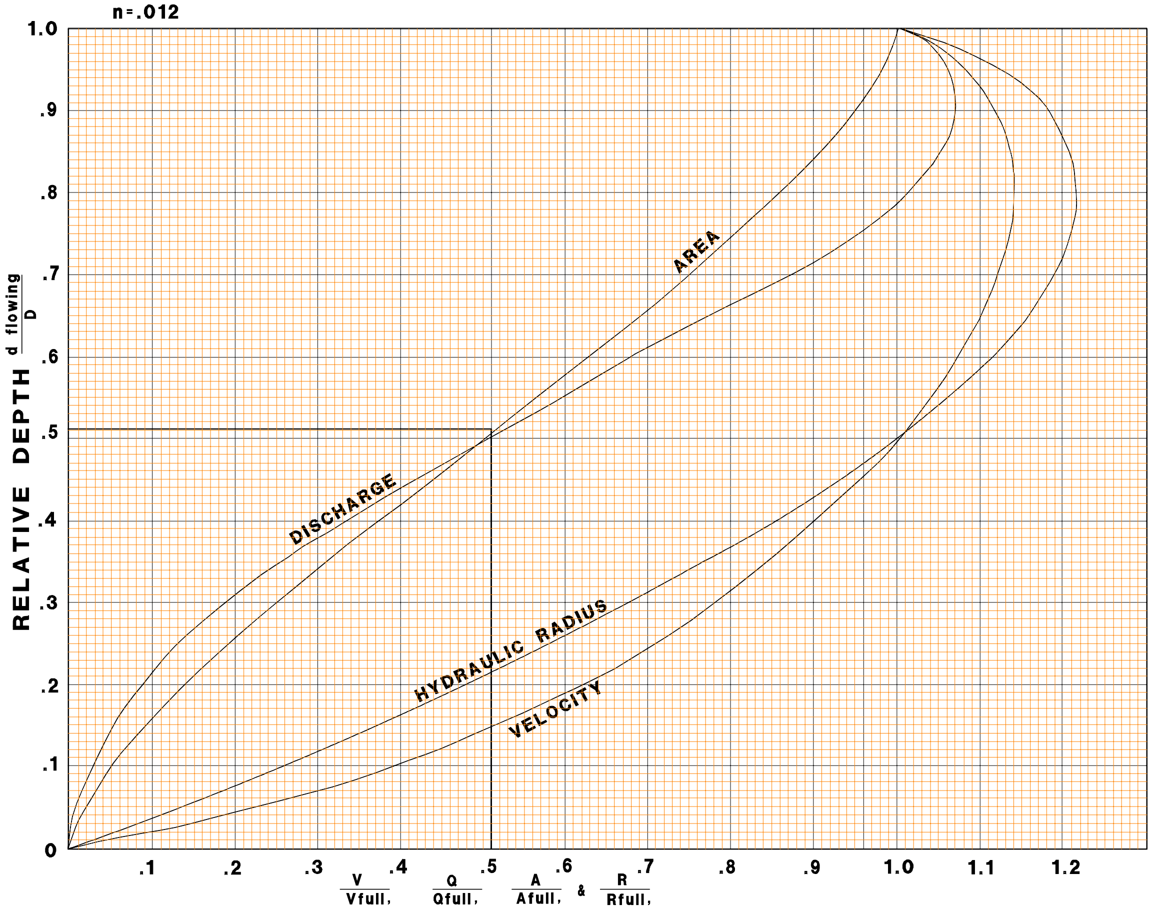
From Hydraulic Design Series No. 3, "Design Charts for Open-Channel Flow"



NOMOGRAPH FOR SOLUTION OF MANNING EQUATION

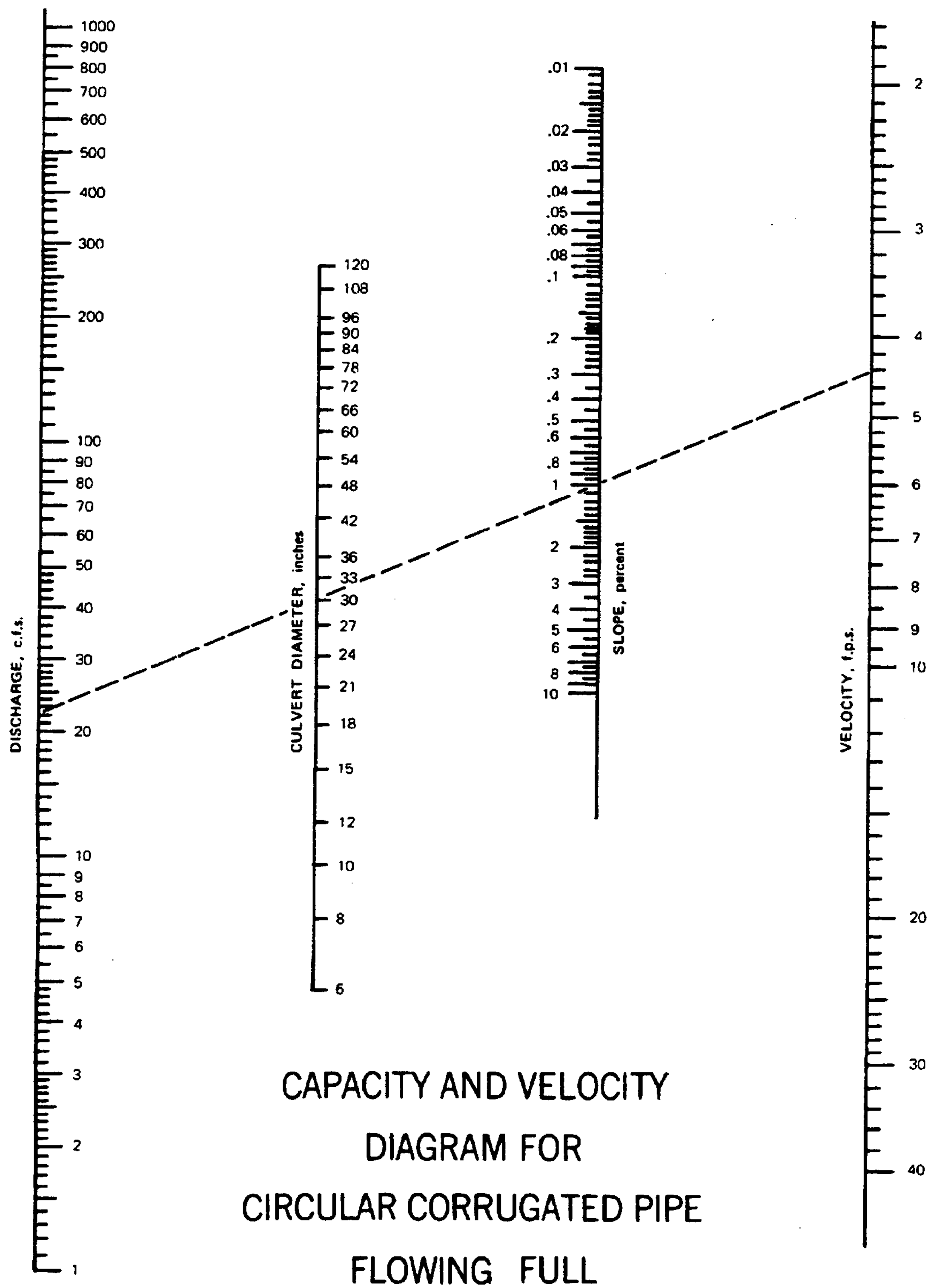
☆ U. S. GOVERNMENT PRINTING OFFICE : 1961 O - 597083

from Hydraulic Design Series No. 3, "Design Charts for Open-Channel Flow"



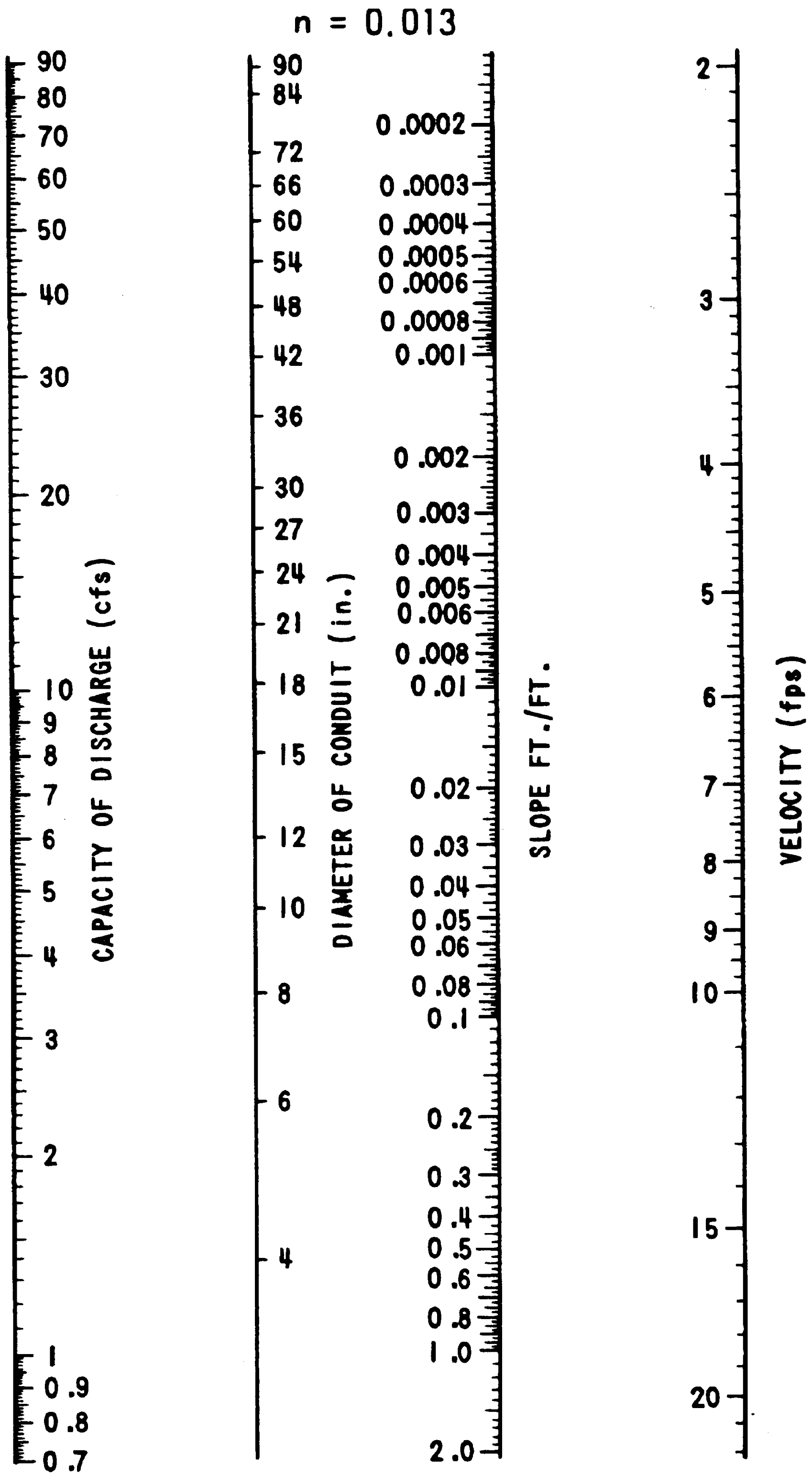
HYDRAULIC ELEMENTS

CIRCULAR SECTION



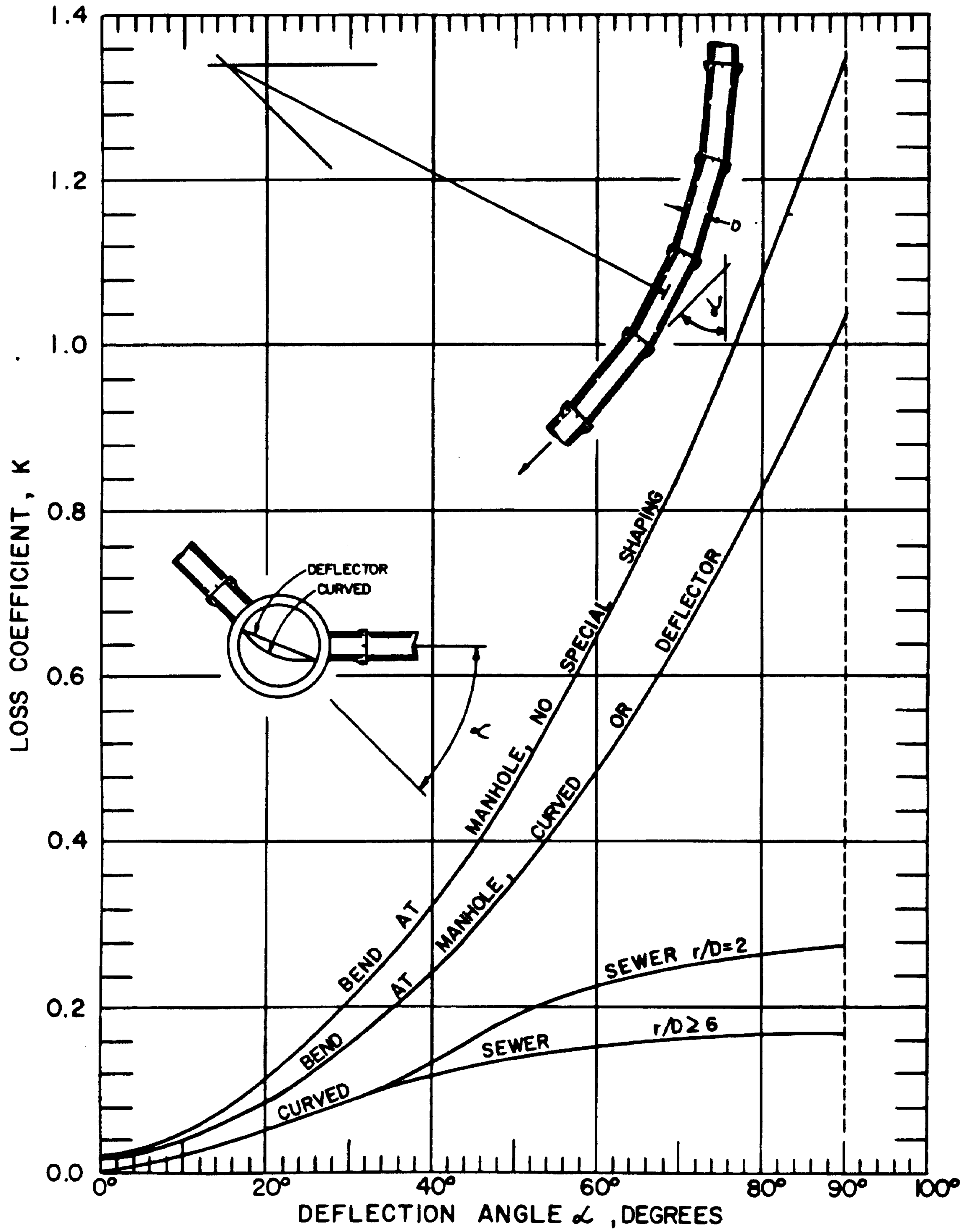
CAPACITY AND VELOCITY
DIAGRAM FOR
CIRCULAR CORRUGATED PIPE
FLOWING FULL
$n = 0.024$

Capacity and Velocity Diagram
For Circular Concrete Pipe Flowing Full

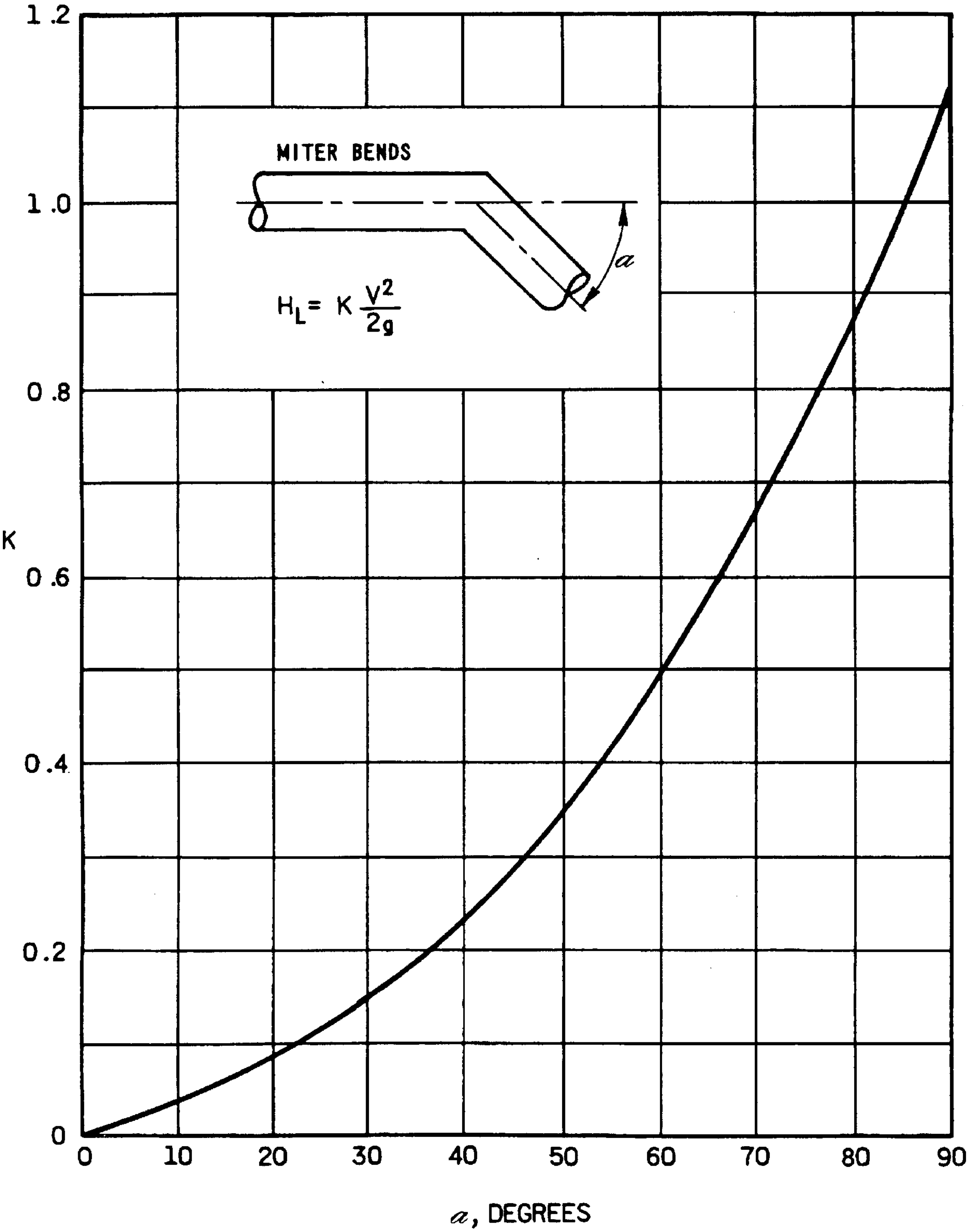


Nomograph based on Manning's formula for circular pipes flowing full in which n=0.013.

Sewer Bend Loss Coefficient



Source: Denver Regional Council of Governments, "Urban Storm Drainage"



LOSS COEFFICIENTS FOR MITER BENDS

Source: Wis. Concrete Pipe Assoc., Curvilinear Alignments for Sewers